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MODELLING OF THE STRESS STATE AND DEFORMATIONS OF APT TESTS

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ABSTRACT

The permanent deformations of unbound materials in low-volume roads have been studied at VTT with two different accelerated pavement tests (APT). The tested structures consisted of different subgrades, layer thicknesses, water contents and pavement materials. The permanent deformations and dynamic responses of various materials were measured both in the vertical and horizontal directions during the tests. The deformation and strength properties of materials were also studied in the laboratory to get reference information. The common objective for these APT was to determine where permanent deformations happen and which factors control the development of permanent deformation. Even though the structures had many similarities, the permanent deformations occurring in the tests concentrated in different layers: in the Low-volume test it was in the base layer and in the Spring-Overload test in the subgrade.

The APT tests were modelled with FEM calculations. The primary aim of these FEM calculations was to get a deeper understanding of why permanent deformations happen in different parts of the pavement. The secondary aim was to find out which is the best way to model stresses. A common way to model a wheel load is to model it as an axisymmetric case with static loading. The best material model for a realistic stress distribution proved to be Mohr-Coulomb's material model. The simple linear elastic model without tension-cut-off property gives high tensile stresses in cases where asphalt layers are thin. The linear elastic material model with tension-cut-off gives better, but not totally acceptable stresses. By using Mohr-Coulomb material model the most sensitive materials and layers for permanent deformations were reliably discovered.

INTRODUCTION

VTT (the Technical Research Centre of Finland) jointly owns with the Finnish Road Administration and VTI from Sweden an accelerated pavement testing (APT) facility called HVS-Nordic. HVS is a linear, mobile heavy vehicle simulator with full temperature control. The loading wheel is either dual or single and the wheel load can vary from 20 to 110 kN. VTT has a test site in Otaniemi with two waterproof test basins where test structures were constructed. Two national test series with low-volume road constructions were carried out during 2000 to 2002. The first test series were part of the

‘Low-volume road research’ -project. The second series were ‘Spring – overload’ tests where the effect of overload was studied under spring conditions. These test series have earlier been reported in Finnra’s (the Finnish Road Administration’s) report series.

This study analyses the deformation and stress results of the modelling of these HVS test structures with Plaxis. Plaxis is a widely used finite element code for soil and rock analyses. This modelling used Plaxis version 8.2. The pavement structures have only occasionally been analysed with finite element programs. One reason for this is the fact that traffic loading is much more complicated than the static loading normally applied in geotechnical problems. Another reason is that the material models in element programs are mainly developed for static loadings, not for repetitive cyclic loading. Because of these reasons the analysis has to be simplified.

OBJECTIVE

The primary aim of the FEM calculations was to get a deeper understanding of why permanent deformations happen in different parts of the pavement. The secondary aim was to determine the best way to model stresses. Since stresses depend on the material model, several kinds of material models have been tested to assess, which material model would give the most reliable stress distribution. This stress-deformation study created a basis for future development of the permanent deformation model.

Both HVS-studies included the determination of material parameters in the laboratory. Defined properties included classification, deformation and strength properties of the unbound pavement materials. The parameters were also defined from the back-calculations of the FWD tests (falling weight deflectometer) and Emu-coil (inductive coil pair for displacement measuring) measurements. The back-calculated and laboratory defined parameters were collected together and analysed to get input parameters for modelling. The construction of the test structures did not succeed as planned, as the structures were in a looser state than the laboratory test samples. Back-calculations revealed this same phenomenon. That is why the input parameters were chosen to be close to the back-calculated parameters.

TESTED STRUCTURES AND PERMANENT DEFORMATION DISTRIBUTIONS

In the “Spring – overload” (SO) research a thinly paved low-volume road structure was tested. The research concentrated on the effects of the overloads to the behaviour of pavement with low bearing capacity under spring circumstances. The second objective of this research was to study the validity of the ‘fourth power rule’ under spring conditions. The total thickness of the structure was 500 mm, in which the subgrade was sand covered with 250 mm layer of crushed gravel as the subbase, 200 mm layer of crushed rock as the base course and 50 mm asphalt on the top (Figure 1). The test wheel was a dual wheel and the wheel was driven on ± 0,3 metre wide range with 0,1 metres intervals between the centre line. The speed of the loading wheel was 12 km/h in both test series. The applied loads were 50 kN and 70 kN and the water table was in the bottom of the subbase. The instrumentation was designed to measure the deformations by several methods so that also the accuracy of the measuring systems could be evaluated (1).

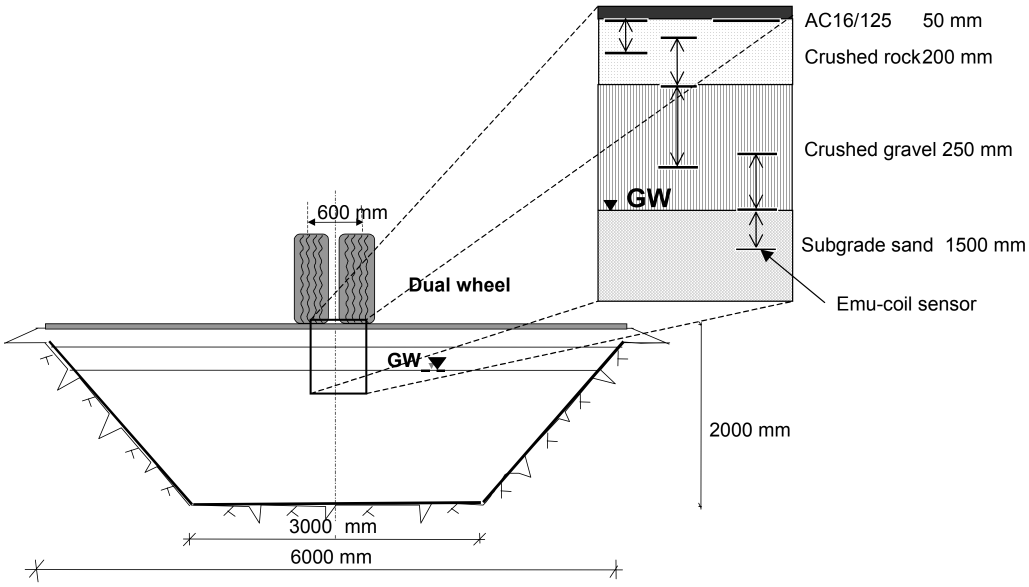

Figure 1. The Spring Overload (SO) structure and Emu-Coil sensors.

The objective of the HVS 'Low-volume' (LV) road research tests was to study the effect of the cross section and edge effects to the structural strength of pavement. In the 'Low-volume' test the total thickness of the pavement layers was 640 mm, which consisted of the 40 mm asphalt layer, 400 mm crushed rock and 200 mm gravel on the subgrade of dry crust clay (Figure 2). The test section was instrumented to measure the changes in stresses, water contents, temperatures and deformations during construction and testing in both the horizontal and vertical directions. The testing wheel was a Super Single wheel, which had the loads of 30 kN, 40 kN and 50 kN. The level of the water was raised from the upper part of the clay to the middle of the base during the test. The response measurements were both dynamic and static (2).

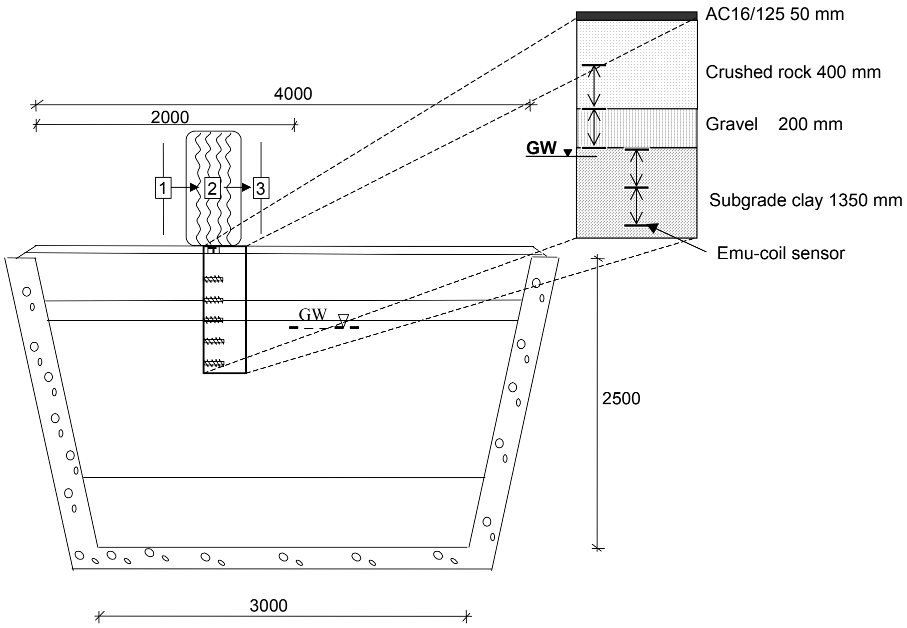

Figure 2. The Low-volume (LV) structure and Emu-Coil sensors.

Figure 3 illustrates the measured distribution of the permanent deformations in the SO and LV structures. The results were quite surprising. In the SO test, where the subgrade was sand, most of the rutting happened in the sand. But in the LV test, where the subgrade was actually softer lean clay, most of the rutting happened in the base layer and subbase. In both cases the test situation was almost identical, with nearly the same water table level, temperature, thickness of asphalt, loading speed and bi-directional loading. The tested wheels and their locations differed to some extent, but the assumption was that these conditions did not have much affect the stress distribution. To understand the reasons for this kind of surprising behaviour the finite element modelling was made.

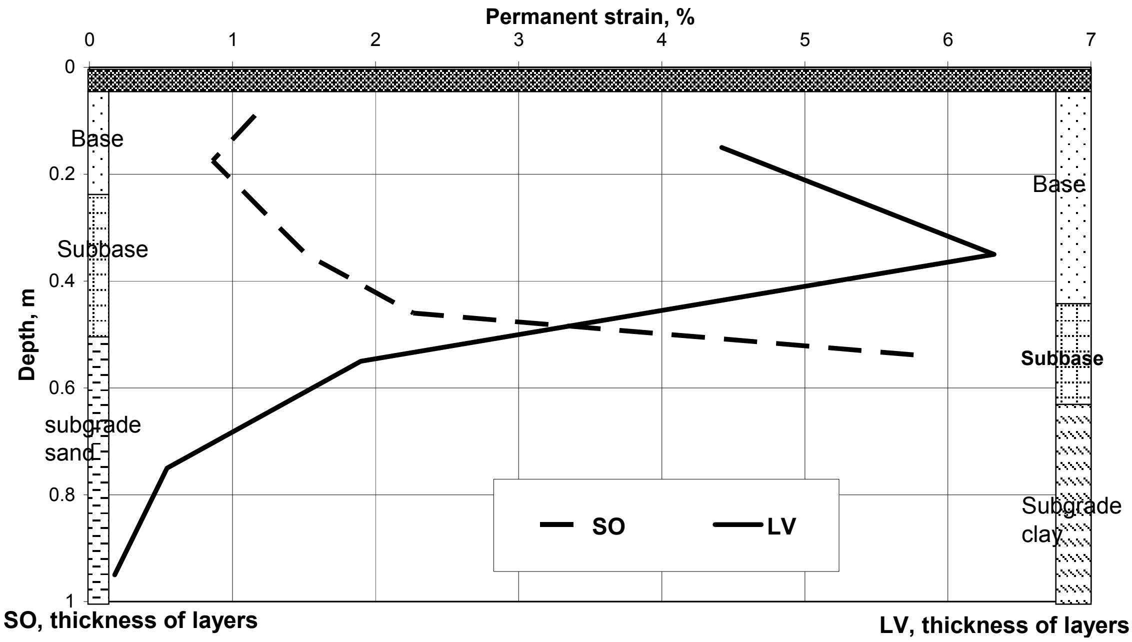

Figure 3. The measured permanent strains in The Spring Overload (SO) and The Low-volume (LV) structures.

MODELLING

The SO structure was modelled with loadings of 398 kPa and 557 kPa and a loading area radius of 0,2 m. The analysis was drained without pore water pressure changes. The input parameters of the calculations are presented in Table 1. Modelling was by a static axisymmetric analysis and the element mesh consisted of triangular elements each with 15 nodes. To simulate the stress dependency of the moduli, the structural layers were divided into sub-layers with the same strength parameters, but different moduli. The axisymmetric analysis was used to get a three dimensional stress distribution. The use of plain strain analysis, where the loading would have been continuous line loading, would have given an overestimation of the stresses and responses. To model the surface load of the dual wheel, the total load was transferred to a circular loading with an average contact pressure. The element mesh and boundary conditions of the SO structure are illustrated in Figure 4.

Table 1. Spring Overload (SO): Input parameters.

Material	**Asphalt**	**Base course crushed rock**	**Subbase Crushed gravel**	**Subgrade Sand**
Thickness, mm	50	200	250	1500
Modulus, MPa	5400	300–220–200	140–90	75
Poisson's ratio	0,3	0,35	0,35	0,35
Unit weight, kN/m^3	25	21,2	22,0	18,0
Cohesion, kPa	-	30	20	8
Friction angle (°)	-	43	44	36
Dilatation angle (°)	-	13	14	6
K_0	1	0,32	0,30	0,42

- not defined

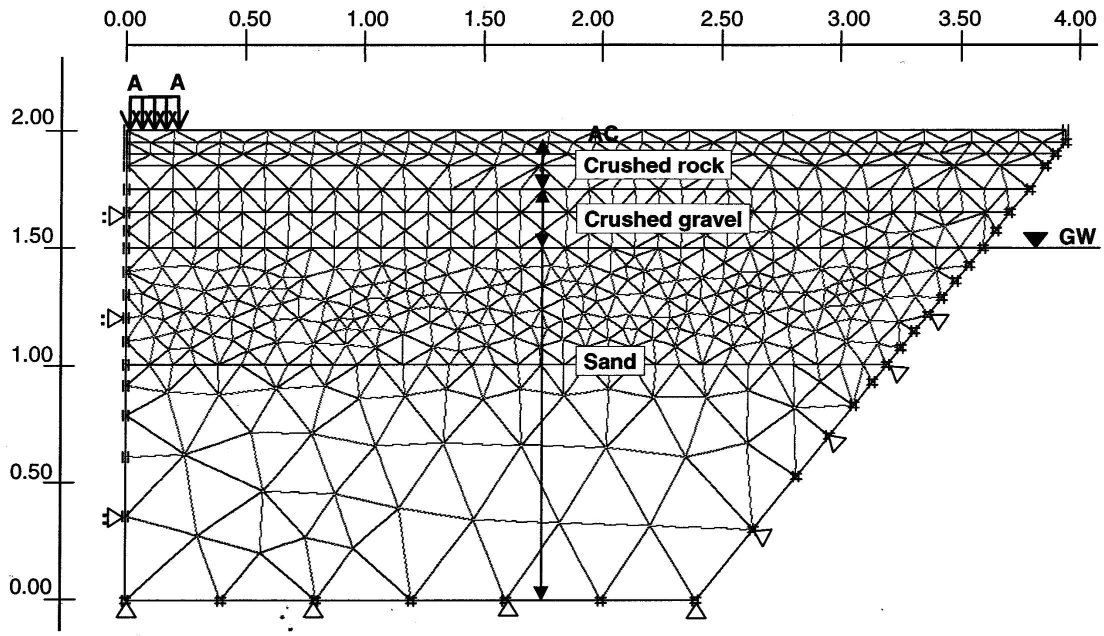

Figure 4. Spring Overload: Element mesh and boundary conditions.

The LV structure was modelled with three contact pressures (424 kPa, 565 kPa and 707 kPa) and with the loading radius 0,15 m to model the Super Single wheel. The input parameters of the calculations are presented in Table 2. The maximum pressures of the tests (557 kPa in SO and 707 kPa in LV) were compared with each other. In both cases only one loading cycle (loading and unloading) was modelled. The calculations were made with the finite element program Plaxis. The asphalt layer and concrete basin were modelled as linear elastic materials. Unbound materials were modelled by using three different material models, which were:

- linear elastic
- linear elastic with tension-cut-off
- linear elastic – perfectly plastic (Mohr – Coulomb).

Table 2. Low-volume (LV): Input Parameters.

Material	Asphalt	Base crushed rock 1	Base crushed rock 2	Subbase Gravel	Subgrade Clay	Bottom Sand
Thickness, mm	50	200	200	200	1350	500
Modulus, MPa	5 400	320–250	150–110	70	10–8	75
Poisson's ratio	0,3	0,35	0,35	0,35	0,35	0,35
Unit weight, kN/m^3	25	21,2	20,5	20	18	18
Cohesion, kPa	-	25	15	9	10	12
Friction angle (°)	-	40	40	36	25	36
Dilatation angle (°)	-	10	10	6	0	6
K_0	1	0,32	0,32	0,4	0,8	0,42

- not defined

In these models the attention was in the stress distributions and in the resilient deformations. All analyses were static. The dynamic analysis was tested with a repetitive half-sin loading, but it was found that the dynamic module of the Plaxis program was not suitable for modelling of traffic loading.

The deformation modulus of unbound material is usually strongly dependent on the stress state, the base and subbase layer were divided into thinner layers with the same strength parameters but with different modulus values. The modelling was started with the parameters derived from the laboratory tests. The measured and calculated resilient responses were compared with each other and material parameters were modified so that the stress distribution and deformations were more realistic (there were no tensile stresses and the deformations were near to the measured ones). Yet the stress distribution changed only slightly because of the modification. The final Plaxis's back-calulated moduli were very near to the back-calculated moduli from FWD (falling weight deflectometer) tests.

One common geotechnical hypothesis about permanent deformations is that the amount of permanent deformation is directly dependent on the stress state; how near the stress state is from the static failure line. This hypothesis is not totally valid in the case of traffic loading, yet it gives a good estimation about the sensitivity of the materials to the permanent deformations. The static failure criteria, which is commonly adapted in geotechnics and with pavement materials is Mohr-Coulomb's failure criteria. The usual assumption is that when the stress state exceeds the limit of 70–75 % of the static failure the magnitude of permanent deformations will start to rise. This study connects the failure ratio to the deviatoric stress ratio. In this study a failure ratio R has been developed, which is presented in Equation 1.

$$R = \frac{q}{q_0 + Mp'}; M = \frac{6 \cdot \sin\phi}{3 - \sin\phi}; q_0 = \frac{c \cdot 6 \cdot \cos\phi}{3 - \sin\phi} \tag{1}$$

where R is the failure ratio

q	deviatoric stress, kPa
q_0	deviatoric stress, when $p' = 0$
c	Cohesion, kPa
M	the slope of the failure line in p'-q space
p'	hydrostatic pressure, kPa
ϕ	friction angle.

ANALYSIS OF THE RESULTS

The results of the modelling are presented in Figures 5 to 13. Three material models were applied as noted earlier. The calculation started with the Mohr-Coulomb material model. In the modelling the modulus of each layer was tested as described earlier and then fixed to some reasonable values. In the linear elastic modelling the same deformation parameters as in the Mohr-Coulomb model were used. The stress states of the subgrade in Mohr-Coulomb analysis in both structures are illustrated in Figures 5 and 7. Figures 6 and 8 indicate where the plastic and tension-cut-off points were situated in the LV and SO structures in the same calculations.

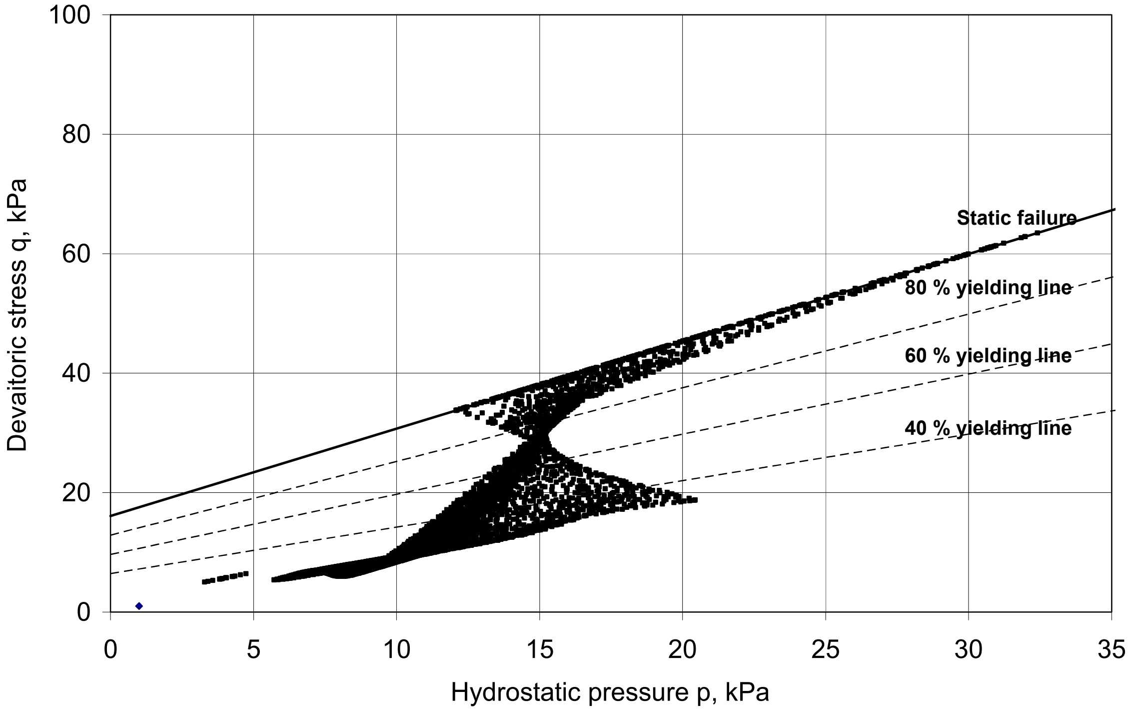

Figure 5. Spring Overload: Stress state (hydrostatic and deviatoric stresses) in subgrade sand.

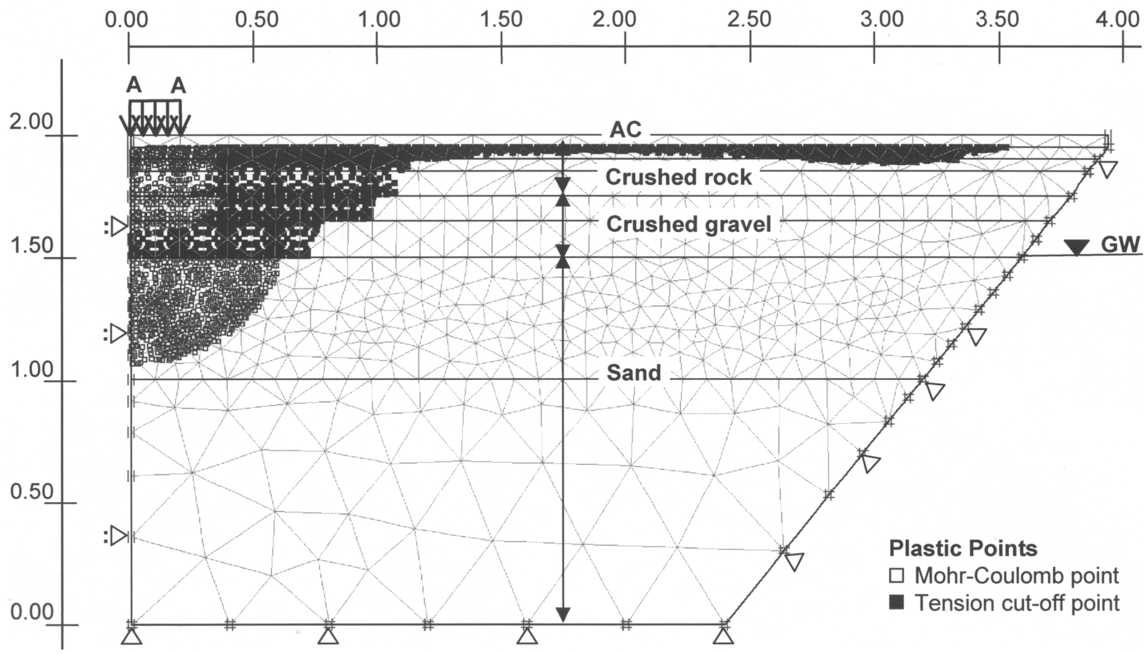

Figure 6. Spring Overload: Plastic and tension-cut-off points.

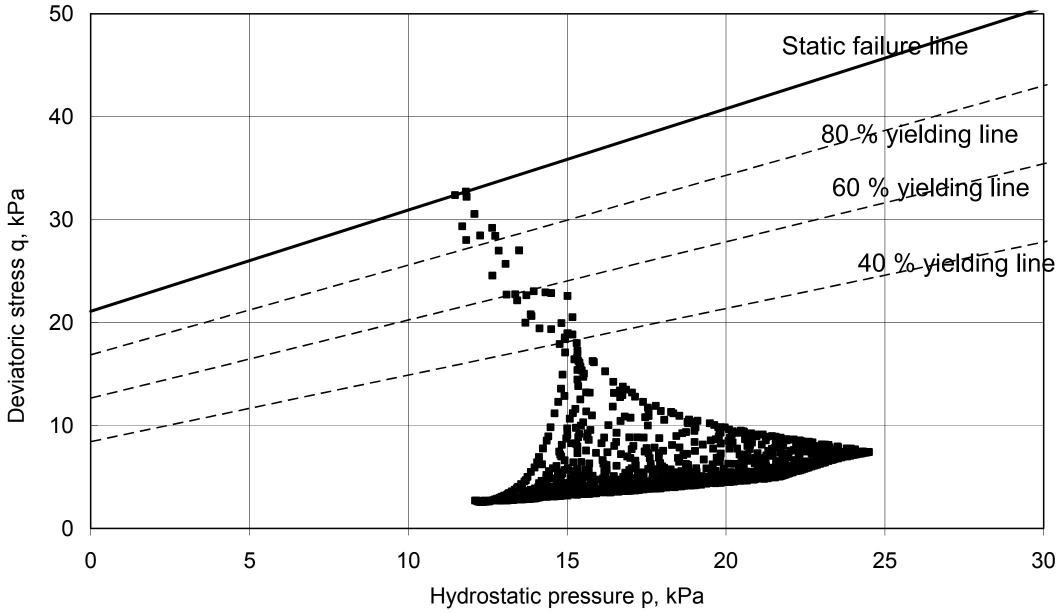

Figure 7. Low-volume: Stress state (hydrostatic and deviatoric stresses) in subgrade clay.

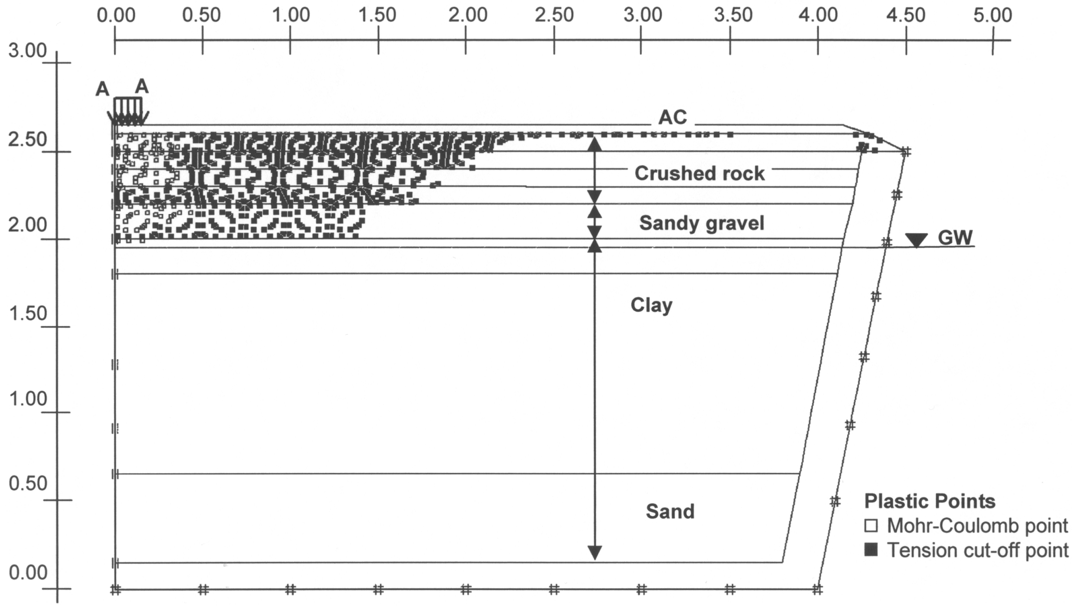

Figure 8. Low-volume: Plastic and tension-cut-off points.

In Figures 9–13 the tested material models are compared with each other to determine, which gives the most reliable results. Figures 9 and 10 present the deflection bowls of different material models in both test structures. Both calculated and measured total deformations under the centre of loading in various materials of SO structure are presented in Figure 11. The failure ratios R (Equation 1) calculated for various material models in the SO and LV structures are shown in Figures 12 and 13. The failure ratio in linear elastic materials has been calculated by using the same strength parameters for each material as in the Mohr-Coulomb material model.

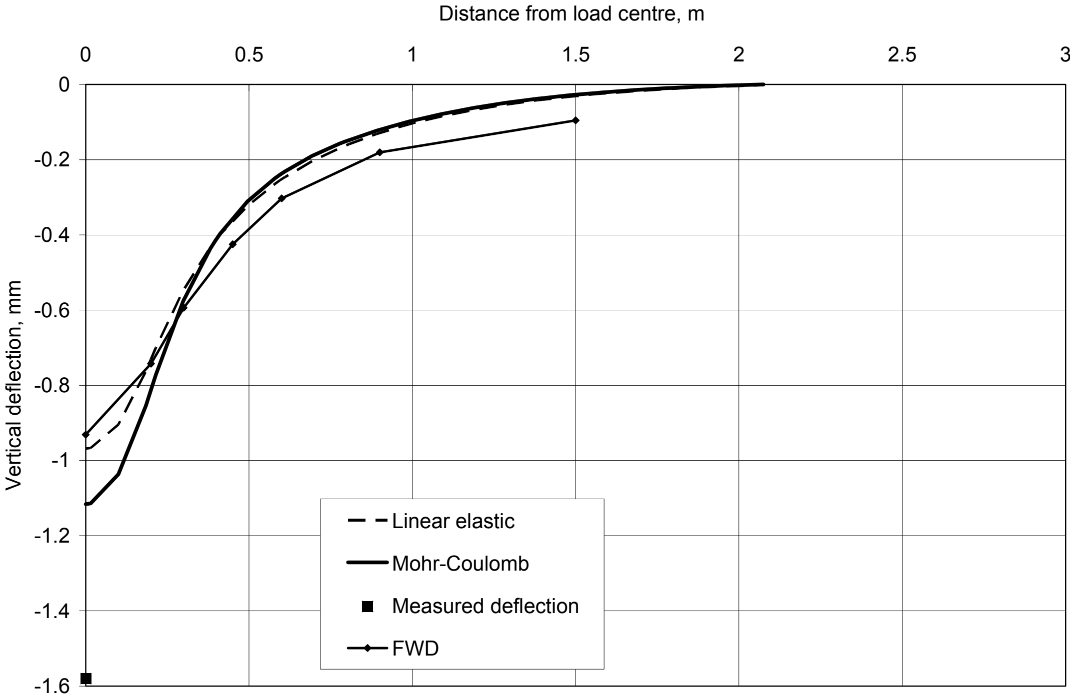

Figure 9. Spring Overload: Deflection bowls of different material models.

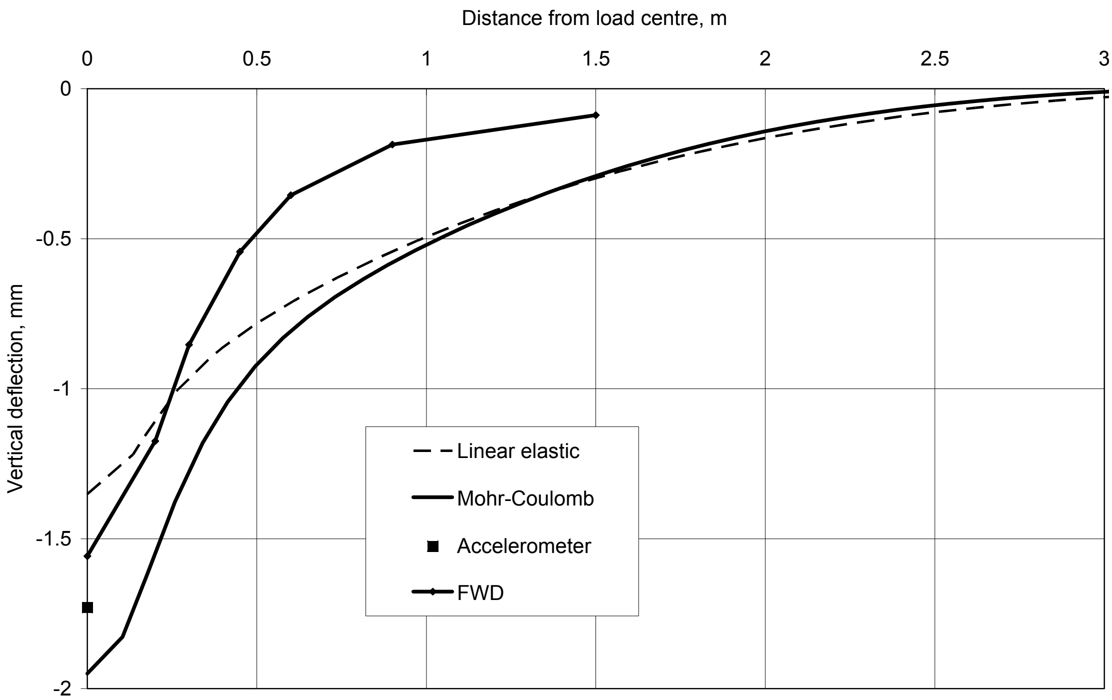

Figure 10. Low-volume: Deflection bowls of different material models.

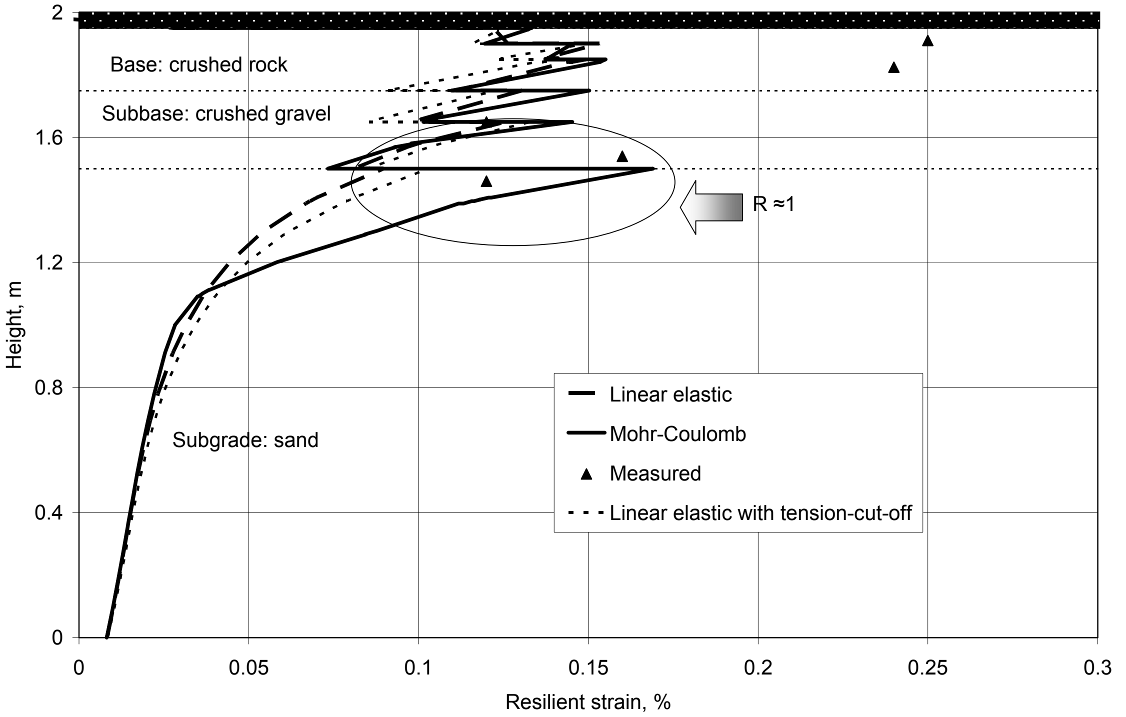

Figure 11. Spring Overload: Total deformations in materials with different material models.

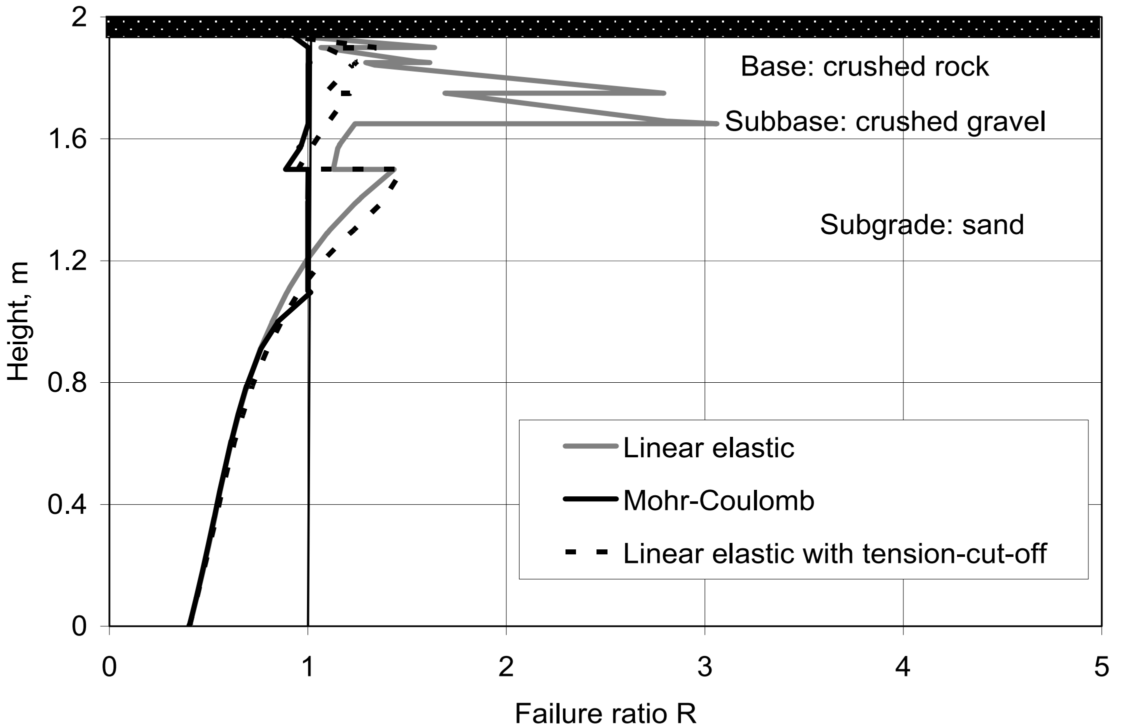

Figure 12. Spring Overload: The failure ratios R of different material models.

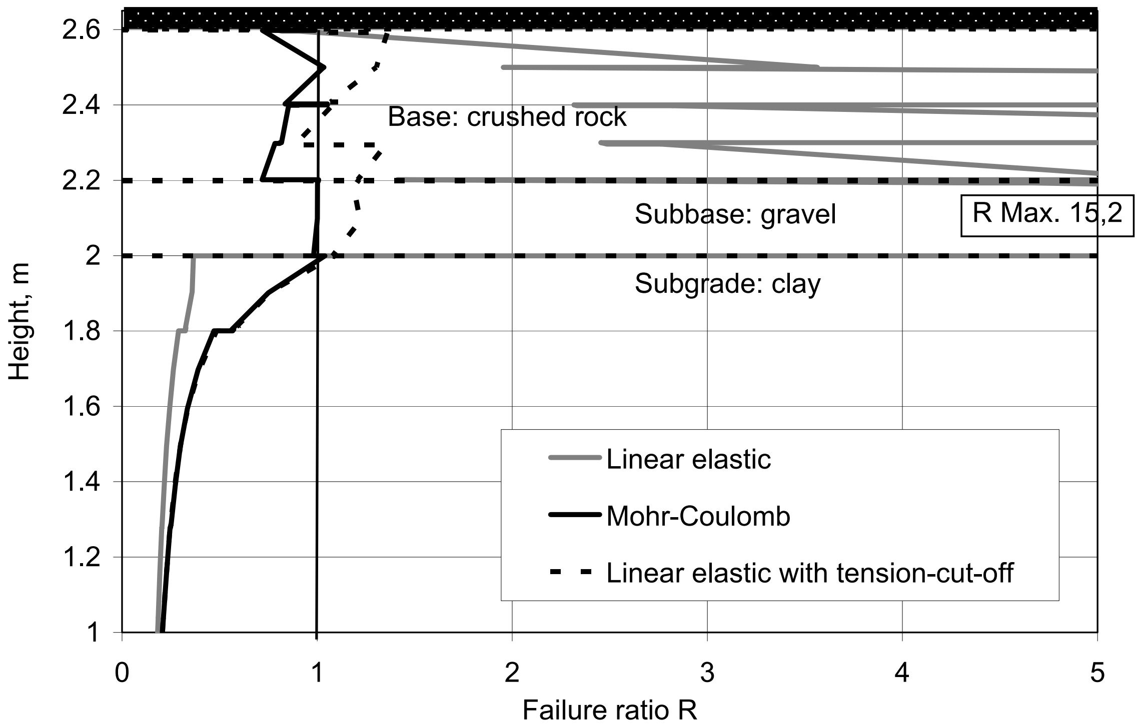

Figure 13. Low-volume: The failure ratios R of different material models.

DISCUSSION

The surprising distribution of permanent deformations in the SO and LV tests demonstrates that it is not easy to predict where permanent deformations will happen. Because of the similarities of the loading conditions, it is obvious that the explanation for the permanent strain distribution can only be the relations between stresses and stiffness (material properties and their thickness). The Plaxis calculations with the Mohr-Coulomb material model confirmed this. In the LV test the base and subbase layers were both near to static failure conditions under loading while the subgrade clay had only a few integration points which had failure ratios over 70 %. When Figures 5 and 7 are compared it is evident, that in SO modelling the amount of points where the stress state was over 70 % is much greater in the subgrade than in the LV test. Figures 6 and 8 show the same result: the plastic points of the SO structure are concentrated to the subgrade. In the LV structure, plastic points were concentrated to the structural layers and there were hardly any points in the subgrade that had reached the plastic limit.

The total deformations in the linear elastic calculations were smaller than in the Mohr-Coulomb model, because in the linear elastic material model the stress state does not have a maximum limit with bigger plastic (permanent) deformations. The calculated deflection bowls of different material models indicate this, as seen in Figures 9 and 10. The deflections of the linear elastic material model with tension-cut-off was situated between the linear elastic and the Mohr-Coulomb models. The applied material models did not have much effect on the width of the deflection bowl in each test. Yet the softer subgrade of the LV structure could be observed by the much wider deflection bowl than in the SO structure. In the SO structure the measured deflections were greater than those calculated. One reason for this was that in the modelling the loading used was an average loading of a dual wheel, not the actual in-situ loading. So the stresses and also the measured strains in the upper part of the structure were much higher in the middle of the loading.

The calculated total deformations, with only a very small permanent part, were not very sensitive to the material model. This was the case unless the failure ratio was near 1 or no larger tensile stress was detected, as Figure 11 shows. This indicates that the calculations of resilient responses are not as sensitive to the used material model as calculations of permanent deformations. The reason for this might be that resilient deformations are governed with mayor principal stresses and permanent deformations are governed by the deviatoric stresses.

The modelling suggested that parts of the structure in these thinly paved examples were in static failure or very near it. The actual situation in real structures is not as alarming as the modelling suggested due to a couple reasons. First the traffic loading lasts only a short time and because the actual strength of the structure is larger than that measured in a short-time shearing test, no bigger deformations usually happen. Secondly, by using more sophisticated models – like non-linear plastic models – where the material deforms more when its stress state is near to failure, a more clear stress state could be created.

The practical maximum value of the failure ratio for the Mohr-Coulomb material model is 1. Linear elastic materials can have extremely high failure ratios because of the tensile stress in the structural layers. The linear elastic material model with tension-cut-off gives much more reasonable results, yet the failure ratio can in some parts of the structure exceed 1. Even if the failure ratio exceeds 1, the basic shape of the ratio (Figures 12 and 13) is nearly the same as in the Mohr-Coulomb model. From these results it is obvious that the linear elastic material model can not produce reliable stress distributions. The same has also been concluded by Huurman (3) in his study of concrete block pavements.

The applied static axisymmetric analysis simplifies the loading and pavement structure quite a lot, because Plaxis can not model cyclic loading nor the three dimensional character of the problem. Another limitation of the modelling is the lack of a material model for cyclic loading with plastic behaviour. Therefore only total strains were examined in this study.

In the HVS tests the portion of permanent deformation in one loading cycle was much smaller than the total deformations. In all tests the permanent deformations have been smaller than 1 % of the total deformations. These analyses are valid only for the cases where permanent strains during one loading cycle are an insignificant part of resilient strains.

CONCLUSIONS

The study indicates that the stress distribution of a pavement can be reliably modelled by finite element program. The chosen material model drastically affects the stress distribution, and also to some extent the resilient deformations. It seems that the sensibility of the materials for the permanent deformations can been evaluated from the modelled stresses. Therefore in the calculation of permanent deformations, it is important to model stress distribution with a better model than a conventional linear elastic material model. By using a linear elastic material model there is a high risk that there will be tensile stresses in the unbound pavement layers. The risk is emphasized in pavement structures which are thinly paved or totally unpaved. These tensions will cause unrealistic stress concentrations with misleading information about permanent deformation sensitivity. The best approach of the three tested methods to model unbound materials is to use the Mohr-Coulomb material model. Reasonable results were also obtained when a linear elastic material model with the tension-cut-off property was applied.

The calculated failure ratio together with the existence of plastic points revealed the most sensitive materials for the permanent deformations. The calculations confirm that permanent deformation distributions are dependent on the geometry, material stiffness and stresses. In future design a reasonable way to verify pavement's sensitivity to the permanent deformation is to model the structure using the Mohr-Coulomb material model.

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